April 2, 2021

TO: A. Byrd, P.E. and R. Washabaugh, P.E.
South Central Region

FROM: T. Allen, P.E. and L. Arnold, Ph.D., P.E.
Construction Division/Geotechnical Office

SUBJECT: I-90, MP 67.5, XL-5479
Snoqualmie Pass Phase 3 Animal Overcrossing
Geotechnical Recommendations

INTRODUCTION

At the request of the South Central Region (SCR), this technical memorandum provides geotechnical recommendations for the Animal Overcrossing associated with the subject project.

This memorandum is one of a series of technical memoranda prepared for specific design elements of the I-90 Cabin Creek Interchange to West Easton Interchange Phase 3 – Add Lanes and Wildlife Bridges Project (Project). Each technical memorandum will be incorporated into the final Project geotechnical report as an appendix. The Project geotechnical report will include detailed information regarding the site and soil conditions and will reference the Project geotechnical data report (GDR), which serves as a central repository for all geotechnical data (boring logs, laboratory test data, etc.). The GDR contains detailed information regarding the geotechnical investigation performed for the Project.

Project Overview

The subject I-90 Cabin Cr I/C to W Easton I/C Phase 3 - Add Lanes Wildlife Bridges project (Project) will widen I-90, straighten curves, stabilize rock slopes, and connect wildlife habitat between approximately MP 64.4 and MP 70.3.

The animal overcrossing of Phase 3 of the Snoqualmie Pass East Project is located on I-90 near milepost 67.5. A site vicinity map is shown in Figure 1. Associated with the project are two precast arch structures and two retaining walls that will allow animals to cross over I-90. The precast arch structures will be approximately 150 feet in length and have spans of approximately 60 feet over eastbound I-90 and 70 feet over westbound I-90. Two retaining walls will support soil that will provide the access and crossing of I-90 over the precast arch structures.

GEOTECHNICAL INVESTIGATION

The geotechnical investigation associated with the Project was planned, coordinated, and managed by WSDOT. The field investigation consisted of a series of subsurface explorations that were completed along the project alignment between October 2016 and October 2020. This work included subsurface test borings, groundwater monitoring, and laboratory testing. Test pits were excavated in several borrow areas during September 2019 to evaluate *in situ* moisture conditions. In addition, historic subsurface information from 1992 to 2016 was reviewed and incorporated into the geotechnical investigation. Please refer to the GDR for a detailed description of the geotechnical investigation.

SUBSURFACE CONDITONS

Soil Conditions

Subsurface information from nine borings was used to develop the geotechnical recommendations for the Animal Overcrossing structure. The locations of the borings are shown on Figure 2 and are shown in Exhibit 1. Please refer to the GDR for a discussion of regional and site geology, a description of the existing alignment, detailed descriptions of the Engineering Stratigraphic Units (ESUs), test boring logs, laboratory test data, groundwater monitoring data, geophysical exploration data, discontinuity mapping, and laboratory testing associated with this portion of the subject project.

The subsurface information used for this study represents conditions at discrete locations across the project site and actual conditions in other areas could vary. Furthermore, the nature and extent of any variations may not become evident until additional explorations are performed or until construction begins. If significant variations are observed at that time, we may need to modify our conclusions and recommendations accordingly to reflect actual site conditions.

EXHIBIT 1: SUMMARY OF FIELD EXPLORATIONS

Boring Name	Northing[1] (feet)	Easting[1] (feet)	Surface Elevation[2] (feet)	Borehole Depth (feet)
H-48-16	706,689.0	1,456,532.7	2,680	31
H-49-16	706,460.5	1,456,566.0	2,651	26
B-16-19	706,563.5	1,456,502.7	2,648	91
B-17p-19	706,565.9	1,456,585.4	2,648	36
B-18p-19	706,419.7	1,456,501.2	2,664	50
B-29-20	706,641.3	1,456,497.3	2,650	21
B-30-20	706,640.2	1,456,584.6	2,648	21
B-31-20	706,550.6	1,456,536.6	2,646	31
B-32-20	706,487.4	1,456,486.7	2,648	32

Notes:
1 Coordinates in Washington State Plane South
2 Elevation in NAVD88

Within the limits of the arch structures, three ESUs were encountered, as described in the GDR.

ESU 1: Existing Fill. This unit generally consists of gravel with sand, silty sand with gravel and well-graded to poorly-graded gravel with silt and sand. Boulders and cobbles are common. Standard Penetration Tests (SPT) blow counts (SPT-N) in ESU 1 averaged 22 blows per foot and were generally between 3 and 41 blows per foot.

ESU 2A: Topsoil. This unit generally consists of sand with gravel and sandy silt with gravel. Cobbles, tree roots, and other small woody debris are common. SPT blow counts (SPT-N) in ESU 2A averaged 12 blows per foot and ranged between 3 and 21 blows per foot.

ESU 2C: Colluvium. This unit generally consists of silty gravel with sand, silty sand with gravel, and sand with silt and gravel. Cobbles, tree roots, and other small woody debris are common. Standard Penetration Tests (SPT) blow counts (SPT-N) in ESU 2C at the Animal Overcrossing site averaged 14 blows per foot and ranged between 7 and 24 blows per foot. Blow counts may have been skewed higher by the presence of coarse-sized particles.

ESU 4B: Alpine Glacial Till. ESU 4B is generally characterized as very dense, silty gravel with sand to very dense silty sand with gravel. Drilling behavior, SPT results, and sample recovery indicate that cobbles and boulders are common in ESU 4B. SPT blow counts (SPT-N) in ESU 4B generally resulted in refusal blow counts (50+ blows for 1 to 3 inches). Blow counts may have been skewed higher by the abundance of coarse-sized particles.

ESU 5B: Felsic Volcanic Bedrock (Pyroclastic/Rhyolite/Andesite/Tuff). ESU 5B is the designation given to the rhyolite and dacite sub-unit of the Naches Formation (Tnr {Tabor} or Enf1/Enm/Enf2 {Cheney 1999}), as well as the altered pyroclastic rock and andesitic bedrock associated with the Taneum Formation (Eaf {Cheney 1999}) and Teanaway Formations (Etf {Cheney 1999}). ESU 5B was encountered in every boring at the Animal Overcrossing site. ESU 5B generally has high RQD values near the ground surface in the area of the proposed arch footings, indicating a relatively intact, unweathered condition.

Borings in the northern portion of the overcrossing (H-48-16, B-16-19, B-17p-19, B-29-20, B-30-20, and B-31-20) encountered bedrock, ESU 5B, within 10 feet of the ground surface. The surficial soils directly over bedrock at these locations consist of ESUs 1 and 4B.

Borings in the southern portion of the site (B-18p-19, H-49-16, and B-32-20) encountered surficial soils and glacial soils near the surface, ESU 2A, 2C, and 4B. ESU 5B was not encountered at these locations until depths of 44 feet and greater. Borings H-49-16 and B-32-20 encountered loose to medium dense soil (ESU 2A and 2C) near the surface. At H-49-16, ESU 2C extends to an elevation of approximately 2,630 feet. At B-32-20, ESU 2A extends to an elevation of approximately 2,635 feet.

Cross sections detailing the subsurface soil conditions at the east and west arch portals are shown in Figures 3 and 4, respectively.

Groundwater

In order to determine the groundwater level at the time of drilling, bailing tests were performed after each boring. Further, open standpipe piezometers were installed in two of the borings (B-17p-19 and B-18p-19). Each piezometer was instrumented and water level readings recorded periodically. Water level information will be included in the GDR. Data collected between October 2019 and June 2020 indicate that the water level varies between an elevation of 2642 feet and 2645 feet near the center and northern portions of the planned overcrossing. Near the southern portion of the planned Animal Overcrossing, the groundwater varies between elevations of 2642 feet and 2653 feet. Based on the nature of shallow bedrock at these locations, it is likely the groundwater at these locations is perched groundwater.

Note that measured groundwater levels represent the times indicated. Fluctuations in groundwater levels may occur due to variations in rainfall, temperature, seasons, and other factors. The contractor should be prepared to deal with perched groundwater encountered in excavations.

SITE SEISMICITY AND GEOLOGIC HAZARDS

Seismic Design Parameters

The ground shaking hazard can be defined in general terms using an appropriate acceleration response spectra and site coefficient, or by using a site-specific procedure.

In the general procedure, the spectral response parameters are determined using the 2014 Seismic Hazard Maps produced by the U.S. Geological Survey depicting probabilistic ground motion and spectral response for 7 percent probability of exceedance in 75 years.

Based on the criteria presented by the American Association of State Highway and Transportation Officials (AASHTO) Guide Specifications for load resistance factored design (LRFD) Seismic Bridge Design (AASHTO Guide Specification) and further analysis, we classified the site soils as Class C. Therefore, the general procedure shall be followed. In accordance with AASHTO Guide Specification, the coefficients provided in Exhibit 2 shall be used.

EXHIBIT 2: SEISMIC DESIGN PARAMETERS

Parameter	Recommended Value
Site Class Based on Soil Conditions	Site Class = C
Mean Magnitude	6.72
Peak Ground Acceleration (PGA) Coefficient of Class B Rock	PGA = 0.23g
0.2-Second Period Spectral Acceleration Coefficient on Class B Rock	S_s = 0.51g
1.0-Second Period Spectral Acceleration Coefficient on Class B Rock	S_1 = 0.15g
Site Coefficient for the Peak Ground Acceleration Coefficient	F_{pga} = 1.20
Site Coefficient for 0.2-Second Period Spectral Acceleration	F_a = 1.30
Site coefficient for 1.0-Second Period Spectral Acceleration	F_v = 1.50
Effective Peak Ground Acceleration Coefficient (g)	$A_s = F_{pga}$*(PGA) = 0.27g
Design Earthquake Response Spectral Acceleration Coefficient at 0.2-Second Period	$S_{DS} = F_a{*}S_s$ = 0.66g
Design Earthquake Response Spectral Acceleration Coefficient at 1.0-Second Period	$S_{D1} = F_v{*}S_1$ = 0.23g

Geologic Hazards

Potential seismic geologic hazards include liquefaction, lateral spreading, flow failure, fault rupture, and landslides. We have evaluated the existing and proposed site topography, soil conditions, and groundwater conditions and consider the risk of these geologic hazards at this site to be low.

GEOTECHNICAL RECOMMENDATIONS

This section of the memorandum presents our conclusions and recommendations for the geotechnical aspects of design and construction on the project site. We have developed our recommendations based on our current understanding of the project and the subsurface conditions encountered by our explorations. If the nature or location of the arch structures, retaining walls, or other aspects of the project are different than we have assumed, we should be notified so we can change or confirm our recommendations. Recommendations are also subject to change based on the results of the pending proposed borings.

Foundations

Arch Foundations

The ESU 1, 2A, and 2C soils are unsuitable for supporting the arch structures. Below the arch footings, where ESU 1, 2A, or 2C soils are present, we recommend these soils be removed to more competent strata, such as ESU 4B or ESU 5B. For the north arch foundations, and the northern foundation of the south arch, existing borings indicate ESU 4B or 5B will be present at the proposed footing elevations.

For the southern foundation of the south arch, existing borings indicate ESU 2A and 2C will need to be overexcavated down to an elevation of approximately 2635 feet at the west end and down to an elevation of 2630 feet at the east end to reach ESU 4B. This overexcavation will also remove ESU 2A and 2C from the proposed mechanically stabilized earth (MSE) wall foundation locations adjacent to the southern arch. Figures 3 and 4 provide the approximate locations and elevations of ESUs at the site.

For the northern foundation of the south arch, borings indicate that some overexcavation will be needed down to an elevation of 2635 feet at the west end to reach ESU 5B. However, note there is some uncertainty for the location of this contact along the length of the footing as rock contacts can change abruptly. We recommend assuming 3 feet of overexcavation for the length of this footing.

Overexcavation should be performed as described in Figure 5 – Overexcavation Details. Where the base of the overexcavation is ESU 5B (rock), backfill material should consist of lean concrete with a minimum 28-day strength of 1,500 pounds per square inch (psi). Where the depth of backfill with lean concrete over ESU 5B is less than 5 feet, the bearing resistance for ESU 5B may be used. If the location of the contact with ESU 5B requires more than 5 feet of fill, the Geotechnical Office should be consulted to determine the appropriate bearing resistance.

Where the base of the overexcavation is in ESU 4B, backfill may consist of lean concrete or Gravel Borrow placed in accordance with Method C, per WSDOT Standard Specification 2-03.3(14)C. Design criteria for both options are presented herein.

The bottom of the footings should extend at least 30 inches below the adjacent exterior grade. Footings bearing directly on rock should be embedded at least 1 foot into the rock unless more is required by the plans, per WSDOT Standard Specification Section 2-09.3(3)C. Where overexcavation is needed to reach ESU 5B, the rock surface should be cut to a firm surface, removing all loose material as described in WSDOT SS 2-09.3(3)C. However, footings placed on backfill over ESU 5B shall not be considered keyed into the rock and rely only on passive resistance from soil adjacent to the footing. Because the footing shall not be considered keyed into the rock, the excavation into ESU 5B may be less than 1 foot if a firm surface with no loose material is achieved. Footing bearing elevations assumed for design were based on plans provided by Contech ("I90 – Phase 3 – Foundations.pdf") on January 28, 2021. The approximate bottom of footing elevations used for design are presented in Exhibit 3. There will be a planned footing step where the bottom of the footing drops approximately 1 foot 6 inches at the approximate midpoint of the planned 156-foot-long footing.

EXHIBIT 3: SUMMARY OF ANTICIPATED ARCH FOOTING BEARING ELEVATIONS

Footing Location	Approximate Bottom of Footing Elevation (feet)
East Portal Northern Archway	2636.5
East Portal Southern Archway	2635.5
West Portal Northern Archway	2638.5
West Portal Southern Archway	2639.0

Exhibit 4 presents nominal bearing resistances for the Strength and Extreme Event Limit States and Exhibit 5 presents nominal bearing resistance for the Service Limit State. The planned footing width has not yet been determined; therefore, we determined bearing resistances based on various effective footing widths. The bearing resistances are valid for effective footing widths between 4 and 33 feet. Footings less than 10 feet wide will experience less settlement than estimated in Exhibit 5. We can provide capacities for other footing widths upon request. We anticipate bearing resistance to be controlled by the Service Limit State.

The settlement (and therefore the Service Limit State) of the arch structures, MSE retaining walls, and earth fill cannot be decoupled. Because of the influence of earth fill behind the MSE walls and arch structures, the Service Limit State is relatively insensitive to either footing width or the base width of the MSE walls. Because the north foundation of the southern arch is expected to bear on rock and the southern foundation is expected to bear on soil, differential settlement between the two southern arch footings is a concern. Two options for reducing the post-construction settlement of the south footing of the southern arch have been provided to the project team:

Option 1 – Construction Staging: Some of the settlement of the arch footing will be induced by the fill on the exterior of the arch. If the fill behind the arch stem wall is placed before the arch is set, the settlement induced by that fill can be ignored for the purposes of differential settlement. One risk associated with this approach is the uncertainty of actual construction sequencing, which can be influenced by other factors.

Option 2 – Lean Concrete Fill: Instead of using Gravel Borrow as fill beneath the south footing of the southern arch, lean concrete fill may be used. We evaluated the effects of 5 feet and 10 feet of overexcavation and replacement with lean concrete on the foundation settlement. Lean concrete should have a minimum 28-day strength of 1,500 psi.

The Service Limit State bearing resistances are provided in Exhibit 5 for settlement limits of 1 and 1.5 inches. A combination of results corresponding to different combinations of the options for settlement mitigation discussed above are provided. Within each ESU scenario, linear interpolation may be used to estimate settlements between the provided values.

Settlement of footings placed on ESU 5B may be presumed to be less than 1/2 inch, in accordance with WSDOT GDM Section 8.11.2.3.2.

EXHIBIT 4: SUMMARY OF BEARING RESISTANCES AND RESISTANCE FACTORS

ESU	Resistance Limit	Nominal Resistance (psf[1])	Resistance Factor
ESU 4B / Gravel Borrow	Strength Limit	30,900	0.45
	Extreme Event Limit	30,900	0.9
ESU 5B	Strength Limit	70,000	0.45
	Extreme Event Limit	70,000	0.9

Notes:
1 psf = pounds per square foot

EXHIBIT 5: SERVICE LIMIT STATE FOR 1-INCH AND 1.5-INCH CRITERIA

Scenario	Nominal Resistance (psf) 1-inch Service Limit State	Nominal Resistance (psf) 1.5-inch Service Limit State
No Settlement Mitigation (Foundation on ESU 4B / Gravel Borrow)	8,900	13,500
Option 1 – Construction Staging (Foundation on ESU 4B / Gravel Borrow)	10,000	15,000
Options 1 and 2 – Construction Staging + Foundation on 5 feet of Lean Concrete	12,000	18,500
Options 1 and 2 – Construction Staging + Foundation on 10 feet of Lean Concrete	15,000	23,000
Option 2 – Foundation on 5 feet of Lean Concrete (No Construction Staging)	11,500	17,000
Option 2 – Foundation on 10 feet of Lean Concrete (No Construction Staging)	13,500	19,000

The T-3 AASHTO Guide Specifications for LRFD Seismic Bridge Design references the Seismic Rehabilitation of Existing Buildings, ASCE/SEI 31-06 as a document that can be used by the structural designer to develop the soil springs for foundation modeling. The Bridge and Structures Office has adopted these procedures for the development of soil springs. Evaluating shallow foundation springs requires values for the dynamic shear modulus, G, Poisson's ratio, and the unit weight of the foundation soils. Exhibit 6 provides design parameters for developing the soil springs. We understand that the elastic parameters for bearing material are needed for the arch structures. These parameters are not provided for ESU 2A and 2C because those ESUs are not recommended for arch structure support.

EXHIBIT 6: ELASTIC PARAMETERS FOR BEARING MATERIAL

ESU	G_0 (ksf)	G/G_0	Shear Wave Velocity (feet/second)	Poisson's Ratio	Soil Unit Weight (pcf[1])
ESU 4B / Gravel Borrow	6,720	0.83	1300	0.3	130
ESU 5B	26,200	0.97	2500	0.2	150

Notes:
1 pcf = pounds per cubic foot

The material surrounding the arch structures will be Gravel Borrow for Structural Earth Walls within the reinforced zone, per WSDOT Standard Specifications Section 9-03.14(4), or Common Borrow outside of the reinforced zone. Consult with Contech on their requirements for backfill around the arch segments outside the reinforced zone of the walls. The peak particle velocity for the design earthquake is estimated to be 0.64 feet per second, based on the attenuation relationship by Joyner and Boore (1988). The maximum free-field shear strain is estimated as the ratio of peak particle velocity and shear wave velocity (using the method described in NCHRP Report 611, 2008). Exhibit 7 contains the free-field shear strain parameters for the anticipated soils surrounding the arch.

EXHIBIT 7: FREE-FIELD SHEAR STRAIN PARAMETERS

ESU	Shear Wave Velocity (feet per second)	Maximum Free-Field Shear Strain (percent)
Gravel Borrow for Structural Earth Walls	1300	0.049
Common Borrow	750	0.085

MSE Wall Foundations

Retaining wall footings are generally expected to bear on ESU 5B on areas north of the south arch. South of the south arch, retaining wall footings are expected to bear on ESU 4B or Gravel Borrow. Figures 3 and 4 provide the approximate locations and elevations of ESUs at the site. Although a small zone south of the southern arch has ESU 2A and 2C near the ground surface, this material will be removed as part of the arch foundation construction.

In some areas, the existing grade may be lower than the proposed MSE wall foundations. Fill placed beneath MSE wall foundations should consist of Gravel Borrow placed in accordance with Method C, per WSDOT Standard Specification 2-03.3(14)C.

For Extreme and Strength Limit States, the nominal resistance values in Exhibit 4 should be used for the design of MSE wall foundations. A resistance factor of 0.65 may be used for the Strength Limit State for MSE wall foundations. For the Extreme Limit State, a resistance factor of 0.9 should be used.

The Service Limit State resistance for MSE wall foundations on ESU 4B or Gravel Borrow should be determined using the first row in Exhibit 5 (the scenario with no settlement mitigation). Settlement of footings and fill placed on ESU 5B may be presumed to be less than 1/2 inch, in accordance with WSDOT GDM Section 8.11.2.3.2.

Lateral Resistance to Sliding

Lateral forces on spread footings will be resisted, in part, by frictional sliding resistance at the base of the footing and passive earth resistance on the edge of the footing. The contributions of both can be added together to provide the total resistance to sliding. It is our understanding that frictional sliding resistance will be determined using AASHTO LRFD Section 10.6.3.4. As such, the friction angles described in Exhibit 8 may be used. A sliding resistance factor of 0.9 should be used for precast concrete footings and a sliding resistance factor of 0.8 should be used for cast-in-place footings.

EXHIBIT 8: SLIDING RESISTANCE PARAMETERS

ESU	Soil Friction Angle, ϕ (degrees)	Tan (ϕ)
ESU 4B / Gravel Borrow	36	0.72
ESU 5B	45	1

Sliding resistance of footings may be resisted, in part, by passive earth resistance, as described in the following section. For passive resistance, a resistance factor of 0.5 should be applied. Ignore the upper 2 feet of passive resistance unless the ground surface in front of the element being considered is paved.

Elastic Parameters for Arch Design

We understand that ConTech plans on using the Finite Element Software CANDE (Culvert Analysis and Design) for the design of the archway. It is our understanding that required input parameters for this program are Modulus of Elasticity and Poisson's ratio. The required parameters are provided in Exhibit 9.

EXHIBIT 9: ELASTIC PARAMETERS FOR ARCH DESIGN

ESU	Youngs Modulus of Elasticity, E (ksi)	Poisson's Ratio, v
ESU 2A/2C	4.2	0.35
ESU 4B	13.9	0.4
ESU 5B	600	0.25
Gravel Borrow for Structural Earth Walls	13.9	0.4
Common Borrow	5.0	0.35

Lateral Earth Pressure

Soil parameters for design of the arch structures are provided in the following table. The soil parameters provided in Exhibit 10 assume the backfill of the arch structures consists of either Gravel Borrow for Structural Earth Walls, Common Borrow, or topsoil. We understand approximately 2 feet of topsoil will be used to top the overcrossing structure. Both Gravel Borrow for Structural Earth Walls and Common Borrow must be placed in accordance with Method C, WSDOT Standard Specification 2-03.3 (14)C. Topsoil placement is assumed to be in general accordance with WSDOT Standard Specification 8-02.3(4). Lateral earth pressure parameters for other materials can be provided upon request.

EXHIBIT 10: NOMINAL LATERAL EARTH PRESSURE PARAMETERS

Parameter	Value		
	Gravel Borrow for Structural Earth Walls	Common Borrow	Topsoil
Soil Friction Angle (degrees)	38	32	30
Soil Unit Weight (pcf)	135	120	100
Active Earth Pressure Coefficient (k_a)	0.22	0.28	0.30
At-rest Earth Pressure Coefficient (k_o)	0.38	0.47	0.50
Passive Earth Pressure Coefficient (k_p)	13.0	6.89	6.11
Seismic Active Earth Pressure Coefficient (k_{ae})	0.30	0.37	0.40
Seismic Passive Earth Pressure Coefficient (k_{pe})	11.5	6.04	5.32

Global Stability Analysis

We performed a global stability analysis on the planned retaining walls using limit equilibrium methods via SLIDE software. Our analyses indicate that an assumed reinforcement length of 0.7H, where H is the height of the wall, meet the factor of safety requirements for global stability specified in the Geotechnical Design Manual (GDM) (WSDOT, 2019). We included a snow load of 320 psf in our analysis. We considered an exposed wall height of up to 44 feet with fill behind the reinforcement zone consisting of Common Borrow. The upper 5 feet of the wall was modeled without full-length reinforcement to account for topsoil and the foundation of the exclusion wall on top of the overcrossing. The soils beneath the MSE wall were assumed to consist of 10 feet of ESU 2A/2C over 30 feet of ESU 4B over ESU 5B. When not founded on the arch spread footings, we assumed a minimum embedment of 3 feet for the retaining walls. Based on our understanding of the project geometry, this represents the most critical anticipated condition for global stability.

Construction Considerations

In areas where ESU 2A or 2C is encountered, it may be too loose for support of foundation elements. The soil must be observed by the engineer prior to placing concrete or rebar in order to assess the suitability of the onsite soils. In areas where loose soil is encountered beneath proposed wall footings and loose or medium dense soil is encountered beneath proposed arch footings, it must be over excavated to more competent material.

Groundwater may be encountered during the excavation to remove the ESU 2A and 2C soils, depending on time of year and precipitation. We recommend the excavation of these soils takes place during the drier late summer and early fall months.

Rock excavation will be required to adequately embed foundation elements into the bedrock, particularly around the northern portion of the site near boring H-48-16. Based on the unconfined compressive strength tests results, the hardness of the near surface ESU 5B rock could be classified as ranging from R3 to R5, moderately strong to very strong rock. ESU 5B may be rippable with large equipment up to approximately 1 foot in depth. If more than 1 foot of excavation is required, blasting may be needed. Additionally, local hard nubs of rock may exist even in the upper 1 foot that may require blasting, pneumatic rock breakers, or hammers to remove. The rock excavation and exposed subgrade for foundation elements must be observed by the Geotechnical Engineer of Record to confirm that the quality and conditions of the rock meet design requirements.

Although not encountered in the site explorations, cobbles and boulders may be present within ESUs 1, 2A, 2C, and 4B. Such large materials could make drilling and/or excavation difficult. Therefore, the contractor should be prepared to deal with large obstructions. In addition, the site soils may also contain relatively clean sand and/or gravel zones, where groundwater may accumulate and be more prone to caving when exposed in a vertical face or encountered in a drilled hole. Contract documents should require the contractor to be prepared to encounter these conditions.

Temporary Excavations

We understand the contractor will be responsible for maintaining the work zone and any excavations for temporary (short-term) stability. The safety of temporary slopes is the contractor’s responsibility, as it is the contractor that will be on site full time and in control of the site. The contractor will also be responsible for temporary shoring (if necessary), and any required dewatering for groundwater or surface water needed to complete the work. In general, drainage trenches and ditches, as described in WSDOT GDM Section 10.3.3 (WSDOT 2019), should be implemented.

Excavations should be made in accordance with all local, state, and federal safety requirements. For planning purposes, the soils across the site are expected to be Occupational Safety and Health Administration (OSHA) Soil Classification Type C.

The stability and safety of open trenches and cut slopes depend on a number of factors, including:

- Type and density of the soil;
- Presence and amount of any seepage;
- Depth of cut;
- Proximity of the cut to any surcharge loads near the top of the cut, such as stockpiled material, traffic loads, or structures;
- Duration of the open excavation; and
- Care and methods used by the contractor.

Based on these factors, we recommend:

- Using plastic sheeting to protect slopes from erosion; and
- Limiting the duration of open excavations as much as possible.

RECOMMENDED ADDITIONAL GEOTECHNICAL SERVICES

During the construction phase of the project, we recommend that WSDOT Geotechnical Office representative(s), in conjunction with the Regional Materials Engineer, provide the following post-report additional services:

- Review the final design plans and specifications to verify that the geotechnical engineering recommendations have been properly interpreted and implemented into the design.
- Attend pre-construction meetings with the Construction Project Engineer and Contractor to review construction-related issues.
- Review Contractor submittals for shoring walls, MSE walls, temporary slopes, and any other geotechnical elements of the Project.
- Observe the geotechnical aspects of construction in the field. This includes, but is not limited to, footing, MSE walls, and fill subgrades, MSE wall construction, drainage, and structural fill placement and compaction.

The purpose of these observations and services is to note compliance with the design concepts, specifications, or recommendations, and to allow design changes or evaluation of appropriate construction measures in the event that subsurface conditions differ from those anticipated prior to the start of construction.

USE OF THIS MEMORANDUM

This report is for the exclusive use of WSDOT and their design consultants and contractors for specific application to the subject project and site. We completed this work in accordance with generally accepted professional practices for the nature and conditions of the work completed in the same or similar localities, at the time the work was performed. We make no other warranty, express or implied.

4/29/2021

Digitally signed by Lorne Arnold

Prepared By: Lorne D. Arnold, PE
Geotechnical Engineer
Hart Crowser, a division of
Haley and Aldrich

Reviewed By: Todd Mooney, PE
Senior Foundation Engineer
Geotechnical Office

Agency Approval Authority: Tony Allen, P.E.
State Geotechnical Engineer

REFERENCES:

American Association of State Highway and Transportation Officials (AASHTO), 2017, AASHTO LRFD bridge design specifications: U.S. customary units (8th ed.) Washington, D. C., AASHTO, 2 v.

Cheney, E.S., 1999, Geologic Map of the Easton Area, Kittitas County, Washington; Washington Division of Geology and Earth Resources; Open File Report 99-4.

Geologic Hazard Maps, 2019, Washing State Department of Natural Resources, https://www.dnr.wa.gov/programs-and-services/geology/geologic-hazards/geologic-hazard-maps#landslides-and-landforms.

Joyner, W.B. and Boore, D. M., 1988, "Measurement, characterization, and prediction of strong ground motion", Earthquake Engineering and Soil Dynamics II – Recent Advances in Ground Motion Evaluation, Geotechnical Special Publication, 20 ASCE, New York, pp 43-102.

National Cooperative Highway Research Program (NCHRP), 2008, Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments. NCHRP Report 611, http://www.trb.org/Publications/Blurbs/160387.aspx.

Petersen, M.D., et al., 2014, Documentation for the 2014 update of the United States national seismic hazard maps: U.S. Geological Survey Open-File Report 2014–1091, 243 p., https://pubs.er.usgs.gov/publication/ofr20141091.

United States Geological Survey, 2019, Quaternary fault and fold database for the United States, accessed August 14, 2019, from USGS website https://earthquake.usgs.gov/hazards/qfaults/.

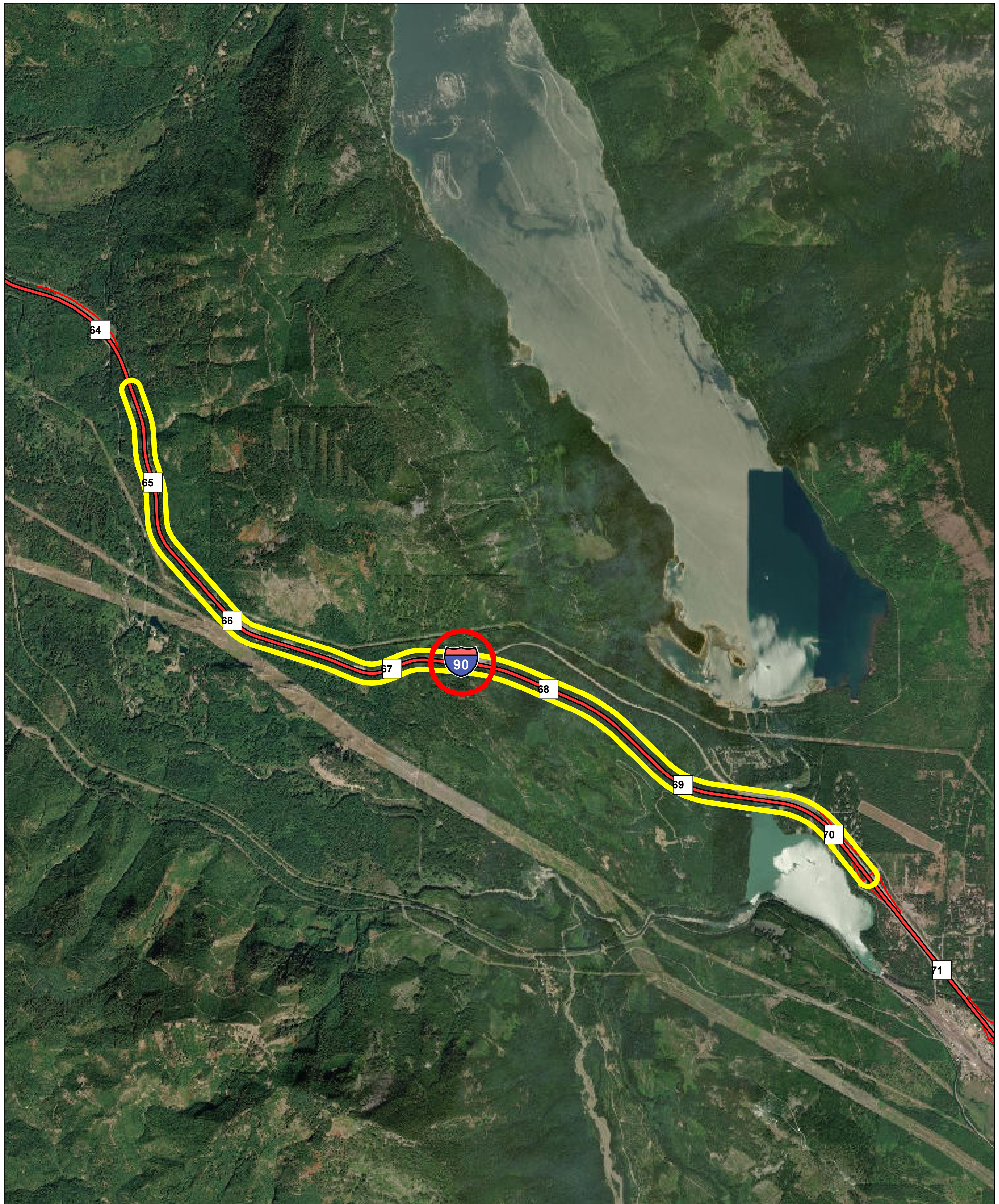
Washington State Department of Transportation (WSDOT), 2019, Geotechnical Design Manual: Olympia, Wash., WSDOT, Manual M 46-03.12, 1 v., July, available: https://www.wsdot.wa.gov/Publications/Manuals/M46-03.htm.

Washington State Department of Transportation (WSDOT), 2020, Standard Specifications for Road, Bridge, and Municipal Construction, Manual M41-10, September, Available: https://www.wsdot.wa.gov/Publications/Manuals/M41-10.htm.

JJW:RBH:DTM:TMA/lda:lik:snb

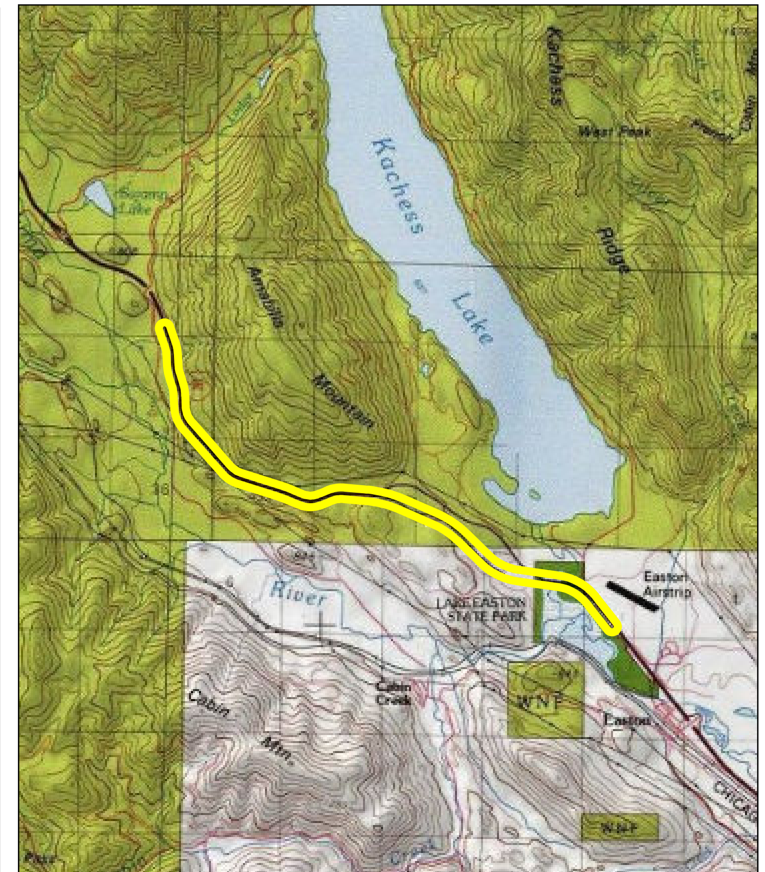
ATTACHMENTS:

Figure 1 – Site Vicinity Map
Figure 2 – Site Map
Figure 3 – East Portal Animal Overcrossing Profile
Figure 4 – West Portal Animal Overcrossing Profile
Figure 5 – Overexcavation Details



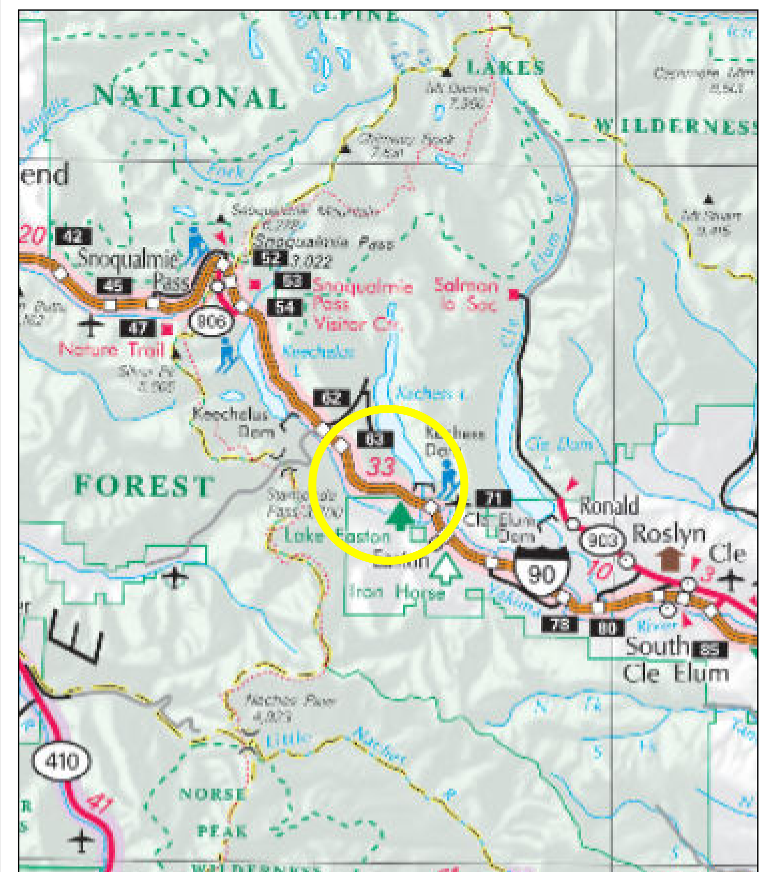
1:48,000
1 INCH = 4,000 FEET

FEET
0 1,000 2,000 4,000 6,000 8,000



1:120,000
1 INCH = 10,000 FEET

MILES
0 0.5 1 2 3 4



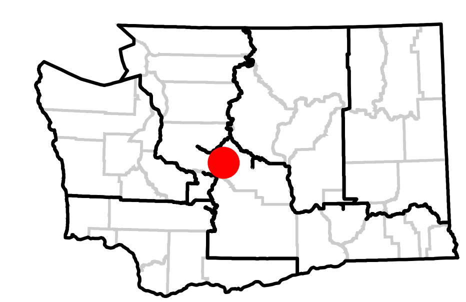
1:600,000
1 INCH = 9 MILES

MILES
0 2.5 5 10 15 20

Legend

- Milepost Markers
- Phase 3 Project Vicinity
- U.S. Interstate
- WSDOT Regions
- County Boundaries (1:500K)
- Animal Overcrossing Vicinity

N



JOB # XL5479 STATE ROUTE 090 MILEPOST(S) 64.40 to 70.30

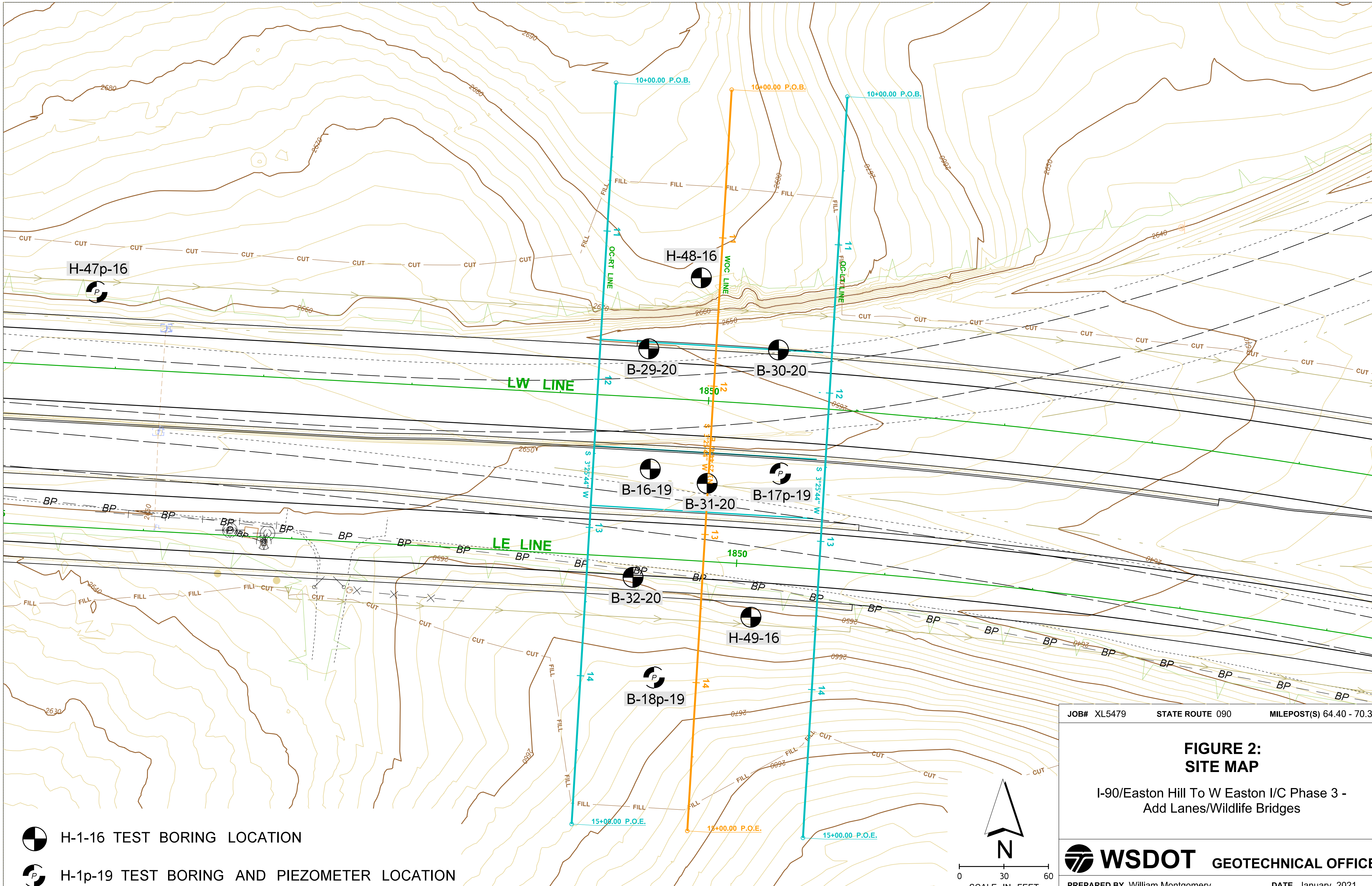
FIGURE 1: SITE VICINITY MAP

I-90 / Cabin Creek I/C to W. Easton I/C
Phase 3 - Add Lanes/Build Wildlife Bridges

WSDOT GEOTECHNICAL OFFICE

PREPARED BY TropleT Date: August 5, 2020

File Location: \\wsdot.loc\ml\Group\Geotechnical\GEOGIS\Projects\SR-090\SR90-XL5479-GT-EastonHilltoWEastonIC\Projects\Phase3\ArcProProject\SR090-XL5479-GT-SnoqualmiePassPhase3.aprx



H-1-16 TEST BORING LOCATION

H-1p-19 TEST BORING AND PIEZOMETER LOCATION

JOB# XL5479 STATE ROUTE 090 MILEPOST(S) 64.40 - 70.30

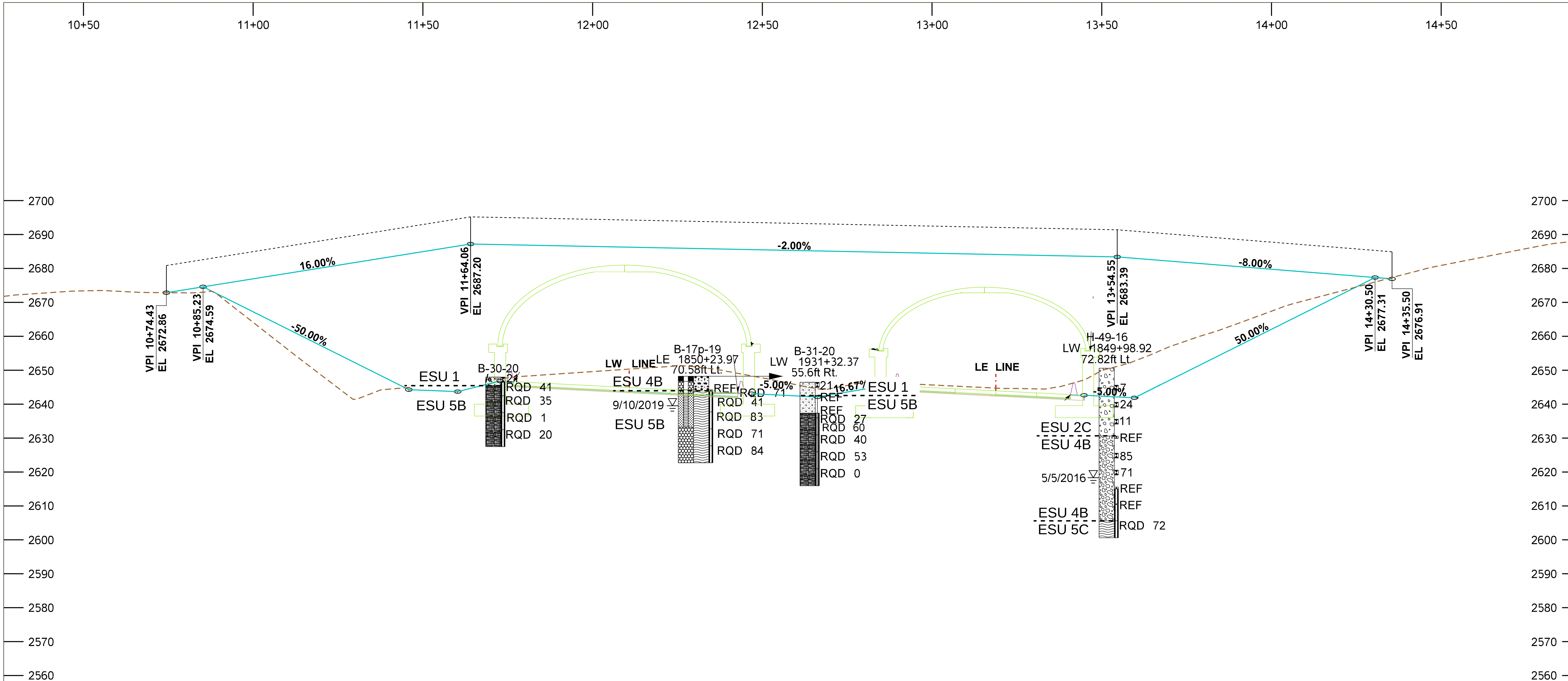
FIGURE 2:
SITE MAP

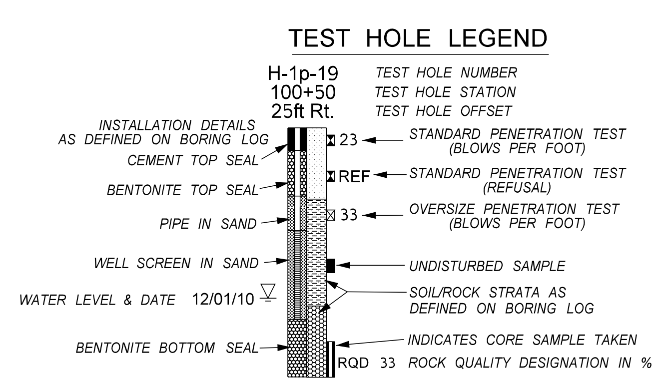
I-90/Easton Hill To W Easton I/C Phase 3 - Add Lanes/Wildlife Bridges

WSDOT **GEOTECHNICAL OFFICE**

PREPARED BY William Montgomery DATE January, 2021

File Location: \\wsdot.loc\ml\Group\Geotechnical\CAE Support\projects\XL-5479 I 90 Snoqualmie pass East Phase 3\wild life OC MP 67_5\XL5479_BP_WOC_MP67.5_BSD.dgn





0 15 30
SCALE IN FEET

JOB# XL5479 **STATE ROUTE** 090 **MILEPOST(S)** 64.40 - 70.30

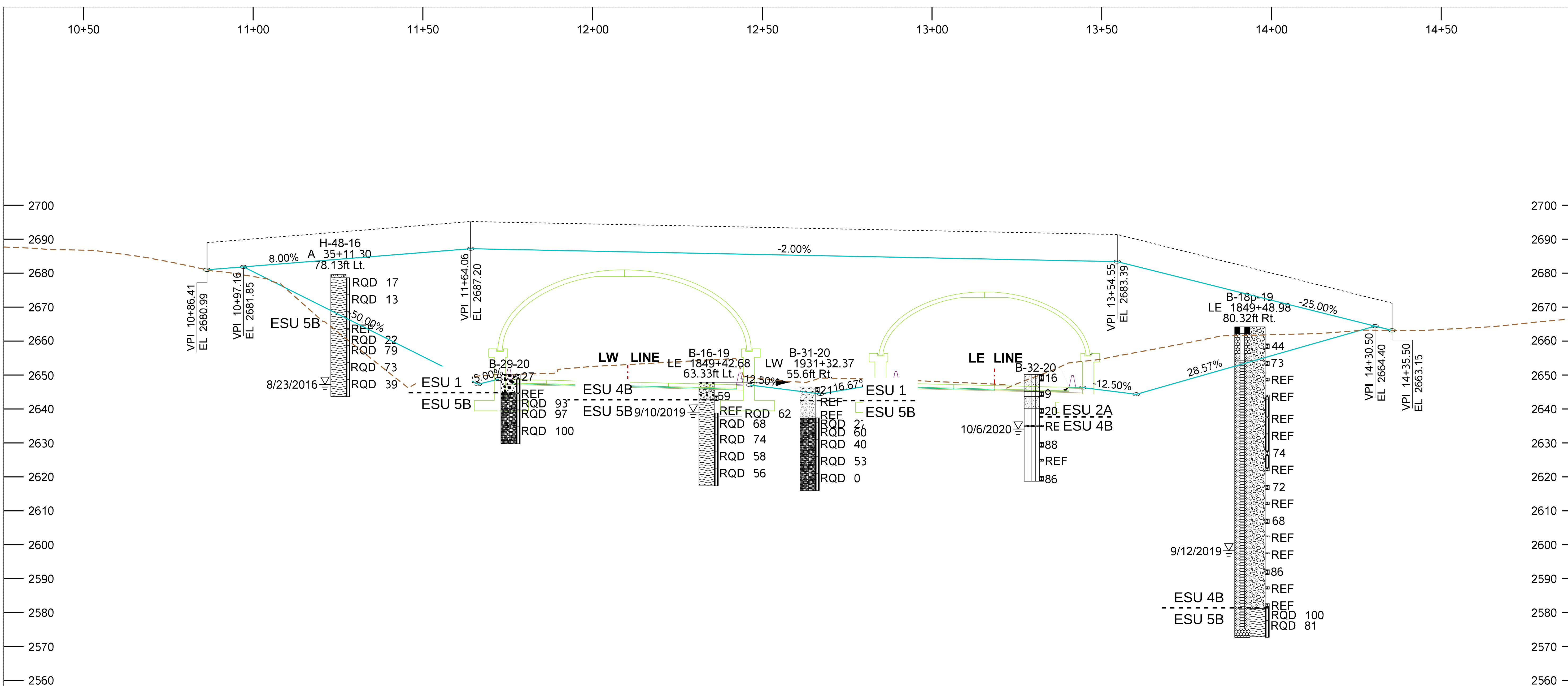
FIGURE 3: EAST PORTAL ANIMAL OVERCROSSING PROFILE

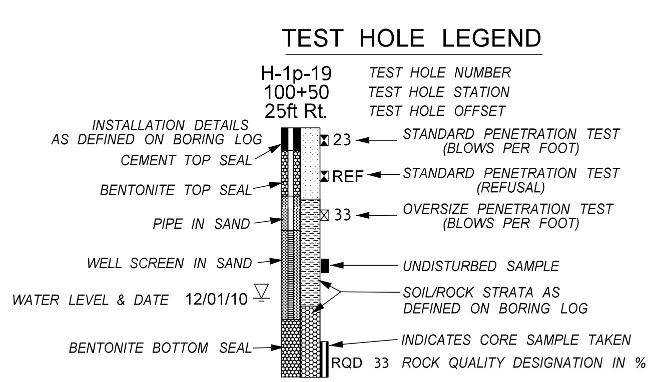
I-90/Easton Hill To W Easton I/C Phase 3 - Add Lanes/Wildlife Bridges

WSDOT GEOTECHNICAL OFFICE

PREPARED BY William Montgomery **DATE** January, 2021

File Location: \\wsdot.loc\ml\Group\Geotechnical\CAE Support\projects\XL-5479 I 90 Snoqualmie pass East Phase 3\wild life OC MP 67_5\XL5479_BP_WOC_MP67.5_BSD.dgn





0 15 30
SCALE IN FEET

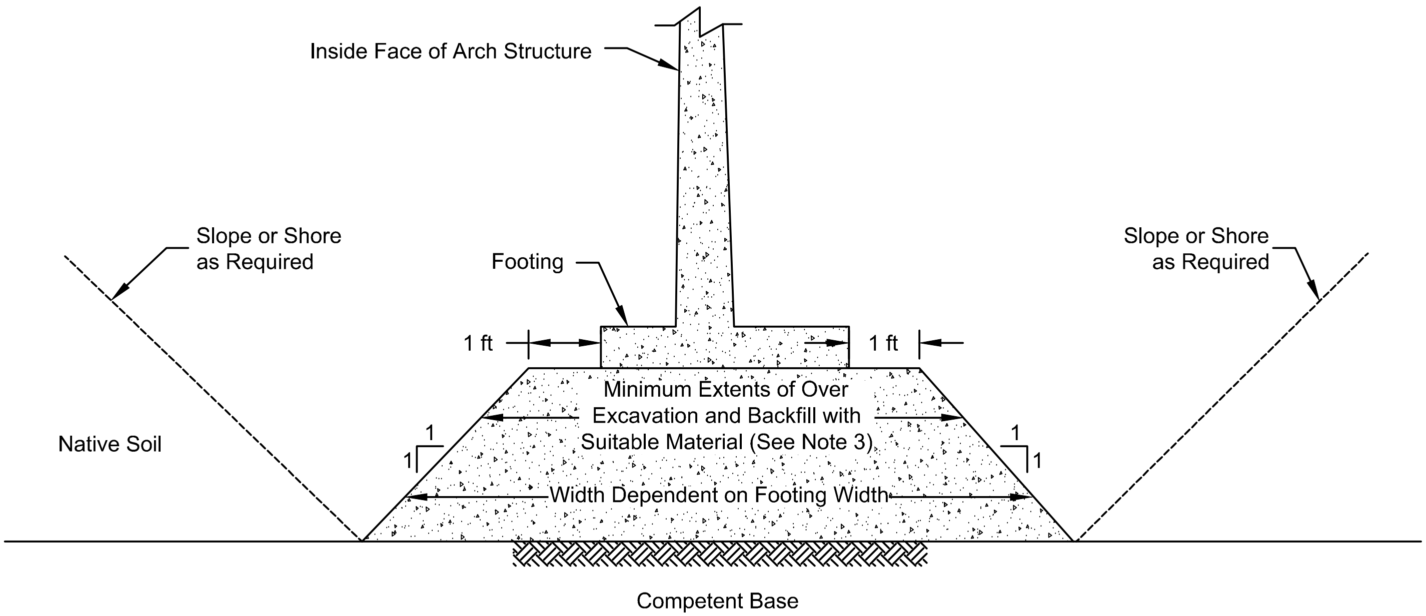
JOB# XL5479 **STATE ROUTE** 090 **MILEPOST(S)** 64.40 - 70.30

FIGURE 4: WEST PORTAL ANIMAL OVERCROSSING PROFILE

I-90/Easton Hill To W Easton I/C Phase 3 -
Add Lanes/Wildlife Bridges

WSDOT **GEOTECHNICAL OFFICE**

PREPARED BY William Montgomery **DATE** January, 2021



N.T.S.

JOB# XL-5479	**INTERSTATE ROUTE** 90	**MILEPOST(S)** 67.5

FIGURE 5: OVEREXCAVATION DETAILS FOR ARCH FOOTINGS

Snoqualmie Pass Phase 3
Animal Overcrossing

WSDOT **GEOTECHNICAL OFFICE**

PREPARED BY Evin Fairchild **DATE** April 27, 2020

NOTES
1) The underlying competent base contact may vary in elevation. Excavate until competent base is encountered. See the report text for approximate contact elevations.
2) Temporary slopes and/or shoring will be needed.
3) Suitable backfill material shall be lean concrete (28-day strength 1,500 psi min) for excavations where the competent base is ESU 5B (rock).Gravel Borrow or lean concrete may be used as backfill material where the competent base is ESU 4B (dense soil). See report text for discussion of fill material and Service Limit State.
4) Gravel Borrow should be placed with compaction method C in accordance with Section 2-03.2(14) of the Standard Specifications.